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ERECTION OF THE MAIN RIVER SPAN, PASSAIC RIVER CROSSING, NEW JERSEY TURNPIKE

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STRUCTURAL DIVISION

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ERECTION OF THE MAIN RIVER SPAN, PASSAIC RIVER CROSSING NEW JERSEY TURNPIKE

Jonathan Jones
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The general design of the Passaic River Crossing was described at the last Annual Meeting (1951) of the Society by Mr. O. H. Ammann of Ammann and Whitney, the designing engineers, and by Mr. Ellis E. Paul of Howard, Needles, Tammen and Bergendoff, general engineering consultants to the New Jersey Turnpike Authority. ("Civil Engineering", January, 1952).

The total length of steel is 6,955 feet. At the lower right or southern end of Figure 1 the structure is seen where it crosses over U. S. No. 1 and then under an arch of the Pulaski Skyway, these being respectively the truck and the passenger car routes toward the Holland Tunnel. Toward the left top is seen the main Passaic River Crossing, whence the structure descends to an embankment approach.

The deck structure is shown in Figure 2 where it passes under the Pulaski Skyway; comprising two three-lane roadways of 36 feet, a 6-foot center mall and two 3-foot safety walks. This deck is carried on longitudinal beam stringers, thence by plate girder floorbeams and floorbeam overhangs to a pair of main girders about 48 feet on centers.

Almost without exception the approach girders were designed as simple spans, resting on concrete piers that comprised two simple shafts and a top cross-member. See Figure 3 Span lengths reached as much as 174 feet, and girder weights as much as 125 tons.

There were many points of interest in the erection of these approaches, including work in close quarters, over main line tracks and under high-tension wires. Space does not allow of describing these procedures; it should however be stated that all except the purely repetitive operations were completely planned in the erection engineering office; dimensioned drawings showing positions of equipment, delivery of material, loads, reaches, sequences and every type of precaution to be observed, were supplied to the resident erection engineer and scrupulously followed.

The justification for the present paper lies in the complexity of the problem of erecting the very unusual spans of the main river crossing; and no more will be said about the approaches excepting insofar as they affected the choice and design of the erection travelers.

Since speed and flexibility of erection were very important under the terms of contract, it was decided that two travelers, each with two booms, as shown in Figure 4, would give greater production than single-boom travelers with the boom capacity required to handle the heaviest girders. The capacities specified in purchasing new travelers were, accordingly:

115	tons	@	65	ft.	reach	for	each	of	two	booms	when	85	ft.	long
100	"	@	75	"	"	"	"	"	"	"	"	115	"	"
70	"	@	96	"	"	"	"	"	"	"	"	115	"	"
45	"	@	110	"	"	"	"	"	"	"	"	115	"	"
15	"	@	135	"	"	"	flat on 20 ft. jib.							

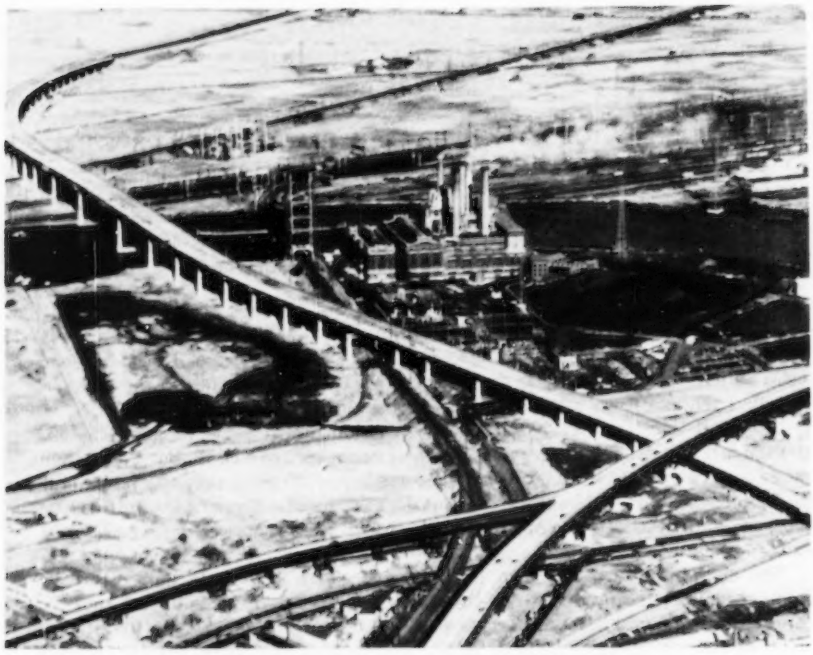


Figure 1. General Airplane View

Figure 2. Deck View Under Pulaski Skyway

Figure 3. Typical Approach Construction

Figure 4. Erection Traveler

Thus the exceptionally heavy girders could be picked by two booms, as shown in Figure 5, while under ordinary circumstances two booms could set two girders in little more time than a single heavier boom could have set one.

Figure 5. Girder Erection

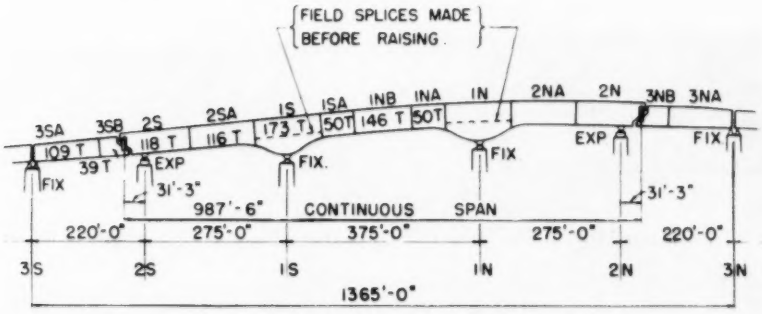
Each traveler therefore was composed of two stiffleg derricks, each with its three-drum-and-swinger, torque converter, air control gasoline hoist of 195 H. P. capacity and line pull of 20,000 lbs. Each such pair of derricks and hoists was mounted on a rectangular platform 48 by 90 feet; trucks under the platform rolled on two rails, one on the top flange of each of the main girders 48 feet apart. Each complete traveler weighed 270 tons.

Coming now to the principal subject, the five main spans of the river-crossing, they are shown in Figure 6 as broken down by field splices for fabrication and erection.

On this diagram it is seen (first), that the main span is 375 feet, or longer than any prior American plate girder span; (second), that this span rests on a fixed shoe at each end, requiring that the concrete piers Nos. 1N and 1S flex under changes of steel length from live load and temperature; (third), that the flanking and continuous spans of 275 feet cantilever an additional 31 feet, to receive an end of the last approach span, making the total length of continuous girders 987 feet 6 inches; and (finally) that the vertical curve over this crossing is unsymmetrical, which doubled the time and cost of many elements of the drawings, the engineering calculations, and the fabrication.

The diagram shows the division by field splice lines into pieces of practical

dimension and weight for shipment and raising; also the system of reference numbering. In general the girders have parallel flanges and are 14 feet deep. But over the two main piers they are increased for the maximum moments to 22-1/2 feet, and so for shipment these sections require the horizontal splices shown by dotted lines. These horizontal splices were connected on the transport barges, making a lift of 173 tons for each girder. The raising of this piece is shown on Figure 5.



MAIN SPAN OVER PASSAIC RIVER AND SHORES

Figure 6.

For fabrication, each girder was laid out on heavy paper on the templet-room floor, in sections of full size with all rivet holes and other details exactly located. The divisions that included the four haunched sections over the main piers were each about 175 feet long. From those layouts, smaller templets were taken off for marking the individual angles and plates of steel. In most of the constituent pieces, occasional holes for bolting-up were sub-drilled according to the templets; but in those pieces which would be on the inner side of the girder when put together, all the holes were thus sub-drilled, as guides.

In Figure 7 the 22-foot deep girder is shown assembled in the shop, including the horizontal splice. The sub-drilled guide holes in the exterior material can be detected, and the lesser number of bolts indicate where the sub-drilled holes pass through all the plies. This girder will now be laid on its side, all the holes will be drilled full-size through the guide holes, and the girder will then be riveted. Perfection of its dimensions, and circular form for its rivet holes, are thus assured.

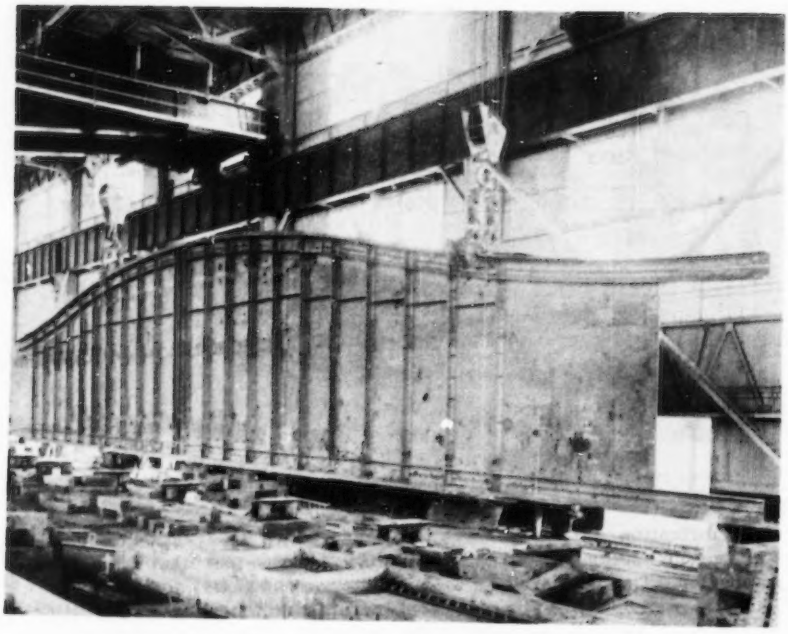


Figure 7. Haunch Girder Assembled

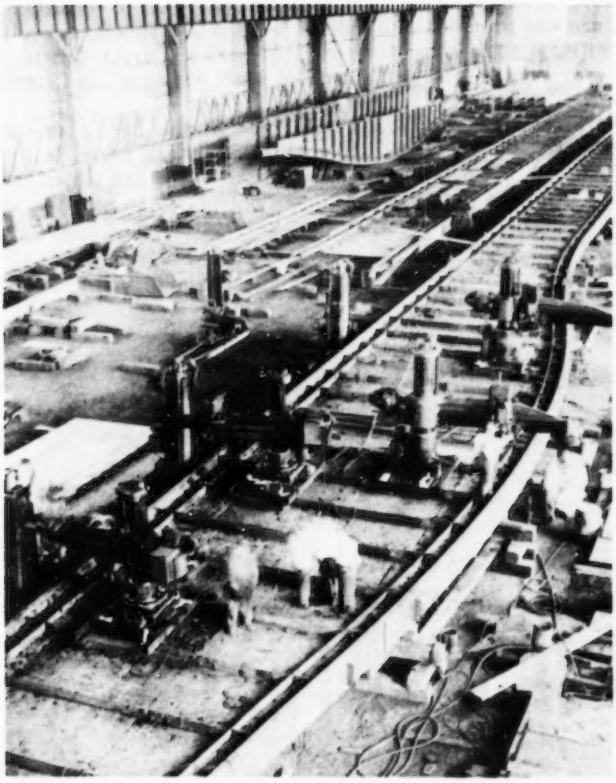


Figure 8. Girder Assembly

In Figure 8 the haunch section of Figure 7 is being assembled at its ends to some of the parallel-sided girders for reaming of the field splices. At this stage the cambered outline of the girder is carefully checked by transit and tape, including the measurement of several triangles determined by the most important working points. In this position all remaining shop rivets are driven, and the field splice material is riveted. insofar as practical, to one of the shipping pieces.

Figure 9 shows the shipment of a 124 ft. length of one of the side-span girders, 2S, the 14-foot depth of which was too great for railroad clearance on flat cars. It was accordingly mounted on the trucks to constitute a "car" in itself, and was provided with special drawbar and standard coupling as seen at the right. The view shows the stiffeners and cross-bracing connections over Pier 2S, and the links from which the suspended girder 3S now hangs.

An interesting feature affecting these links is, that due to the roadway at this point being on a horizontal curve, there is a small angle, in plan, between the two girders. The upper and lower portions of the link plates are planed on opposite tapers in thickness, and bored normal to the resulting outer surfaces; the pins are therefore at the requisite angle in plan, one with the other.

These preliminaries serve to introduce the engineering studies for erection. These are a joint enterprise of the Erection Department and the department

Figure 9. Shipment of 124 ft. Girder

of the Chief Engineer; the former primarily concerned with expediency, cost, and the detailing of appurtenances; the latter with stress calculations and degrees of risk, design of temporary materials, and the deformations and necessary manipulations of the structure in the various stages toward completion. These functions considerably overlap; through cooperation they result, after consideration of many schemes, in a final scheme considered the best, involving the positions of field splices, the positions of temporary supports, practical traveler positions, delivery route for materials and similar general decisions.

The deformations and manipulations referred to, form one of the most interesting and at times rewarding, experiences of the structural engineer. The changes of size and shape of a great steel bridge, while imperceptible to the sidewalk superintendent, are in reality, matters of inches, and of prime importance to the erector. If a group of rivet holes in a given piece does not come into the intended exact alignment with the same group in the connecting piece, even by a quarter of an inch, and cannot be put there by available and safe manipulations, then there is going to be grief. But when by forces pre-calculated and provided the inertia of the structure is overcome, and the great mass of steel moves responsively through the pre-calculated movements, into the intended relation with its fellow, then the planners naturally sense a renewed satisfaction with their place in human affairs.

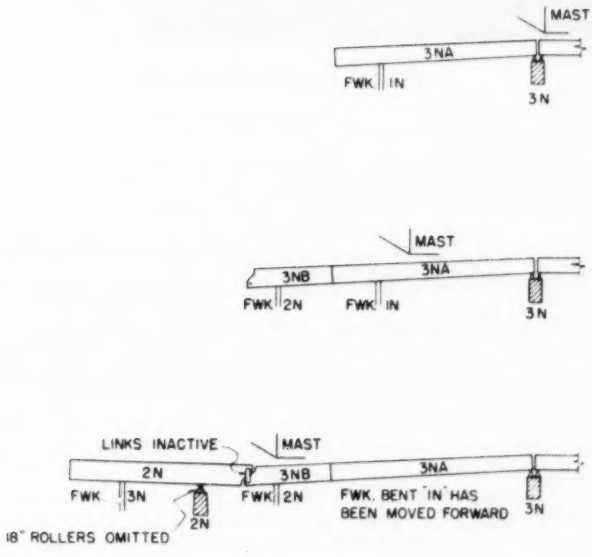


Figure 10. Start of Erection

Figure 11. Erection at Pier 2N

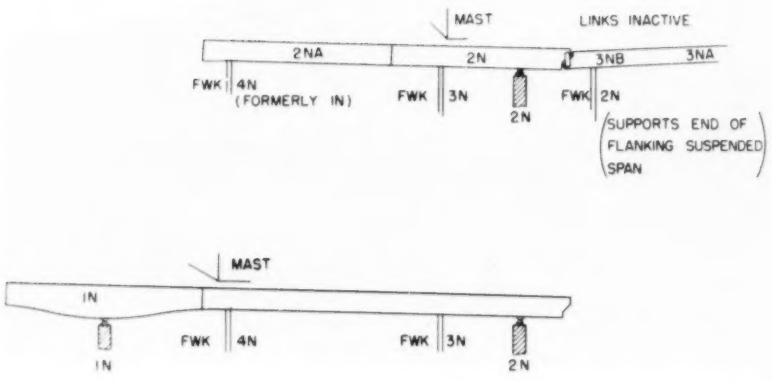


Figure 12. Erection of Span 2

The first step was the calculating of the elastic constants of the girders, over small increments of length, due to their rapid and substantial variations in section. The similar data calculated by the consultants in checking their continuous design, gave us a useful check; but had to be independently repeated so that the chosen increments of length would fit our field splice positions and our positions of temporary supports.

Starting with the prescribed vertical contour of the completed bridge, hypothetical deflections were calculated as for the removal of all weight. The resulting contour (higher, naturally, at the middle of the main span and in other places lower, than the final or "normal" contour) was the "cambered" or "no load" outline; or that to which the girders should be placed when on their sides in the shop for reaming of splices and for riveting.

Girders thus dimensioned, if they could be erected as completed pieces on the four piers and allowed to deflect under first their own weight and then the weight of the concrete roadway, should conform exactly to the intended normal contour.

Since unfortunately they had to be erected in nine pieces each, and since their own weight plus that of the floor steel was great enough to cause material deformations, their tendency would be to deflect to different angles, one piece with the next, from the angles at which the splices had been reamed; meaning that splices would stand open at top when closed at bottom, or vice versa.

The last splices to be made, since erection would proceed simultaneously from each end, would be those at the ends of the center piece, 1NB, Figure 6. At all splices prior to these two, the difficulty just mentioned could be overcome by bringing the splices together and bolting them up while the forward piece was still carried by the traveler boom; then lowering it to its support with elastic bending at the splices thus treated. For the last two splices, however, which would be made simultaneously at the two ends of the final section, it would be necessary for the ends of the two receiving sections, then cantilevered from Piers 1N and 1S and naturally drooping, to be brought up to such elevation and angle that the final section would connect, and when connected and released from the traveler falls would assume the proper contour for dead load-less-floor slab. To ensure this it was necessary to control, or manipulate, the placement and support of each of the four prior pieces coming in from each end. Thus the geometry of the closing position dictated a forced geometry for the prior stages.

A study was made to determine approximately the position of the travelers and the hitches for picking the center section, which was about 115 ft. long between web splices and weighed about 146 tons, including flange splice plates. It was determined that the most practical method of entering the closing girders would be from the side, as the web splice material could be tack riveted to the main material, the overhanging upper flange splice plates supported by a runner line from the boom, and the lower flange plates allowed to droop, to be finally pulled up by long bolts. As the flange splice plates of the closing girders would extend about 26 feet beyond center line of the web splice at each end, they could be shipped loose, and be field riveted while on the barge before erection.

Now having an assumed weight to be picked, and a position of the traveler which the cantilevered sections of the main span could support, deflection calculations were made to determine the necessary girder elevation at Pier 2. This was found to be about 20" below normal at the south end and 15" at the north end. 17" could be made available by removing the 18" rollers on Piers

2 and dropping the anchor bolts temporarily into the pier. By controlled lowering on the falsework near Pier 2S so that some load would be on the falsework and some on pier 2S, it was found that the ends of the cantilevers could be properly positioned for entering the closing girders. No load was assumed on the hinge; for when the time should arrive to jack up Spans 2 to replace the rollers, the calculations indicated that the Span 2 girder could not carry the reaction of the suspended span on the cantilevered length of over 70 feet from the falsework bent to the link. Carrying the load of the suspended span on an independent bent would keep the loads on each of the two bents close to that required in earlier stages of erection. In order therefore to prevent transfer of stress at the hinge point, the holes in the upper ends of the links were slotted vertically 3" and the suspended span held 1-1/2 inches high, with the erector checking after each major pick to see that the pin stayed in the middle third of the slotted hole.

While therefore the calculations preceded from mid-span backwards, and might be mistaken for plans to take the bridge down rather than put it up, the field operations are best shown in their actual sequence. In the upper and middle sketches of Figure 10, sections 3NA and 3NB are successively erected, with the traveler in the respective positions noted "Mast", and their erection requires two temporary erection bents. Together they constitute girder 3N, which is the 190 ft. simple span designed to be supported on Pier 3N at its outer end and on a pin link at its inner end.

In the lower sketch of Figure 10 section 2N has been erected with the traveler standing over forward bent 2N, and the rearward bent 1N has been taken out for re-use. Connection of suspended span 3N to cantilever arm has been made at the link, but no load is being transferred through it. The (exaggerated) droop of the profile pictures the fact that a nest of 18" rollers has been omitted from the bearing of girder 2N on Pier 2N. Using the shoe on Pier 1N as a fulcrum the omission of the rollers is going to lift the steel in the main span.

Figure 11 shows exactly the foregoing situation, though at Pier 2S, on the opposite side of the river. The original bent 1S is being taken out by a crane; bents 2S and 3S are in place. The traveler has just erected girder sections 2S and is filling in the floor steel. It may be detected that there are no rollers under the steel shoe on Pier 2S. The pin link is evident, and the fact of a floor break is indicated by the double brackets.

In the upper sketch of Figure 12 Span 2N is completed in customary fashion, Section 2NA overhanging the falsework bent 4N, which was originally 1N and was taken down and brought forward.

No shear is being passed through the links between Spans 2N and 3N. Across their top flanges there has been installed an adjustable connection. This stabilizes the structure longitudinally to the fixed shoe back at Pier 3; also it can be manipulated to secure fair holes when the shoe at Pier 1 is reached.

In the lower sketch of Figure 12 the main span has been reached, and the deep section 1N has been spliced and raised. The raising of 1N off the barge, with the load taken on two booms, is shown in Figure 5. Figure 12 exaggerates the degree to which 1N is forced to slope upward by the omission of the rollers on Pier 2N.

Figure 13 shows one of the sections 1N raised onto its shoe, being the first of the pair. Because of its great depth it would be unstable if both booms let go of it and went for the companion pieces. The figure shows that a splice was therefore introduced into the permanent sway frame, and about one-fourth

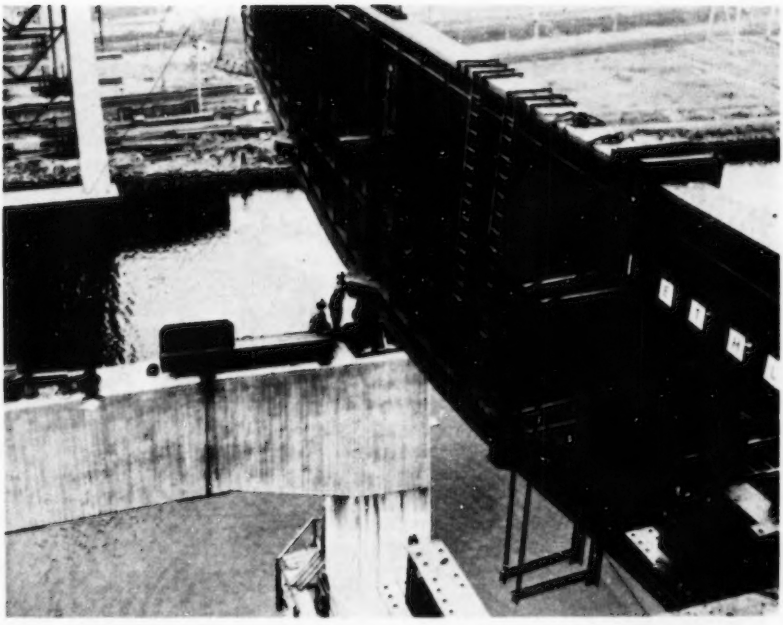


Figure 13. Section 1N in Place Bracing for Stability

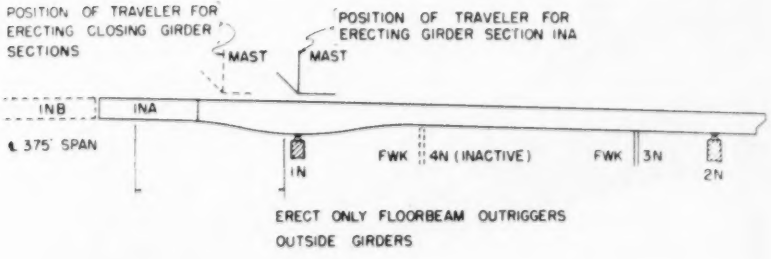


Figure 14. Completion

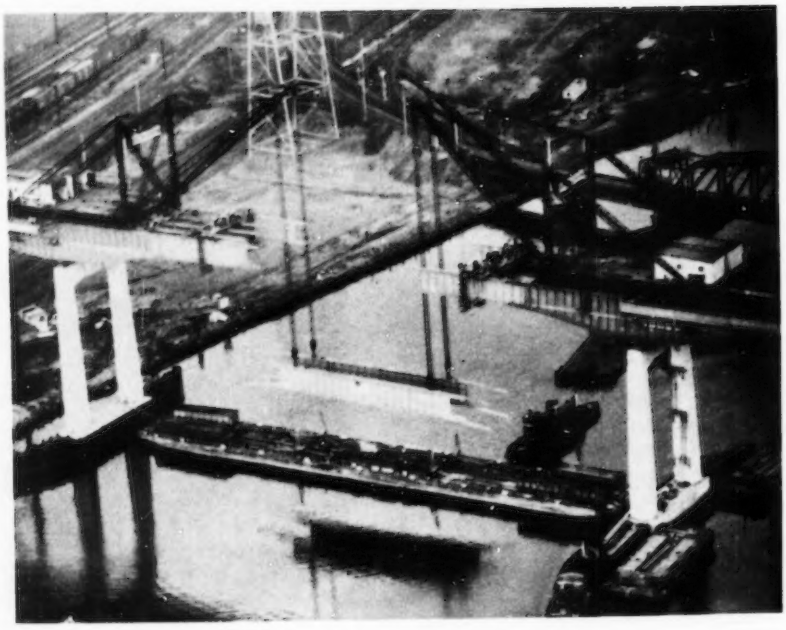


Figure 15. Picking the Closing Girder

Figure 16. Sections 1NB Up and Inside

of its total length was used as a temporary brace between the top of the girder first erected and the pier.

In Figure 14 the traveler standing over the main river pier 1N has erected the 65-foot section 1NA and has advanced to a position 50 feet from the main pier, where it will stand to erect, with its opposite number, the 115-foot, 146-ton closing section 1NB. From the pier to the end of section 1A, all floor steel not required for stability or convenience, is omitted to reduce the cantilevered weight. In exaggerated fashion the Figure indicates that the projecting end has been raised to the point desired.

In the airplane view Figure 15 the two closing girders have been brought in on cars and car float, and are being hoisted simultaneously, with no cross connection and at somewhat less than their final center-to-center distance. This simultaneous pick had been decided upon because (1) the effect of warping the span from unsymmetrical picks would be eliminated; (2) equipment and man power would be available and (3) the channel would need to be blocked only once.

Figure 16 gives some conception of the difficulty of making the final splices. It is obvious that the splice material is so long that entry by sideways entry was imperative. Each flange at this point included a side plate on each side of the main web, then four angles forming an H, then additional plates over the vertical legs of each angle pair. The clearance between these latter plates and the inner horizontal surfaces of the outstanding legs of the flange angles between which they lay, was so small that a slight error in the angular position of the cantilevered girders would have made the entry of this material impossible. This angular position had however been put under control by the provisions made for raising or lowering the falsework in the flanking spans (Spans 2), which would cause the main span cantilevers to rotate about Piers 1. The existing positions were checked by instrument, and entry to the splices was determined to be feasible, before the two closing sections were picked off the barge.

It has been mentioned that the main 375-foot span is fixed to the piers at both ends. In order that one pair of girders might be retracted (at main pier 1S) so the final entry could be made, longitudinal motion was provided for at the top of the shoe, where a plane surface adaptable for sliding existed in the design, and where jacking brackets could readily be attached to the girders. Of the jacking force to be exerted, all except that due to friction on the sliding surface would obviously react against the concrete pier and deflect it. The Engineers placed a limitation on this deflection corresponding to 250 kips thrust per shoe; to design the jacks and brackets 10 percent of the vertical load was added and 200-ton jacks selected. The sliding surfaces were machined particularly smooth and were well greased, thus justifying the friction coefficient cited. Jacks in opposing pairs were provided, so that the girders after final splicing could be re-centered on the shoes as required.

Figure 18 shows the structure after the splices had been entered and were being prepared for riveting.

Parenthetically, this view naturally raises a question as to the stability of these girders, under their own great weight and without lateral support for a length of 250 feet. This had been checked by an adaption of the method of Karl deVries (Trans. A. S. C. E. Vol. 112, 1947, pp. 1245-1271). The fact that they were stable followed from the fact that top and bottom flanges were equally substantial. Girders with top flange reduced with a view to "composite action" with a concrete deck are far less stable for erection, and frequently require temporary stiffening before they can be swung as simple spans.

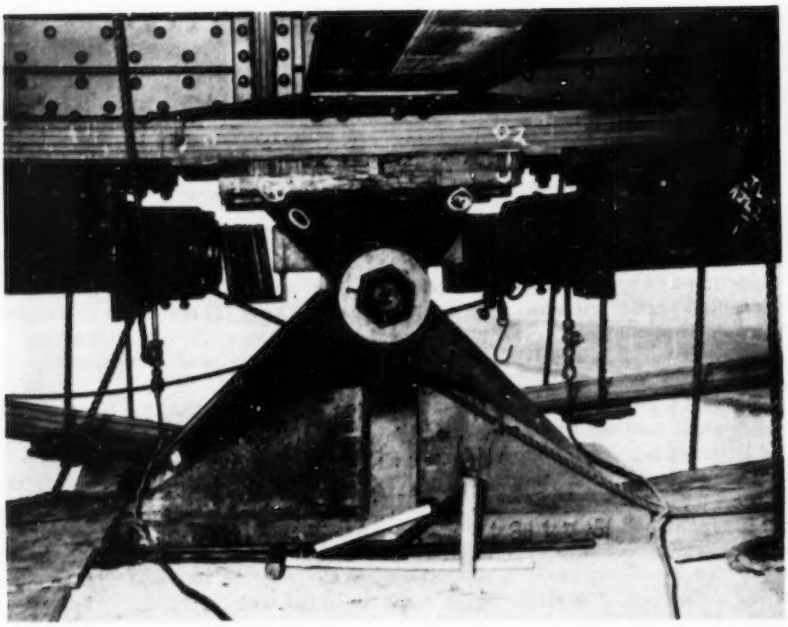


Figure 17. Jacking Arrangements at Pier 1S

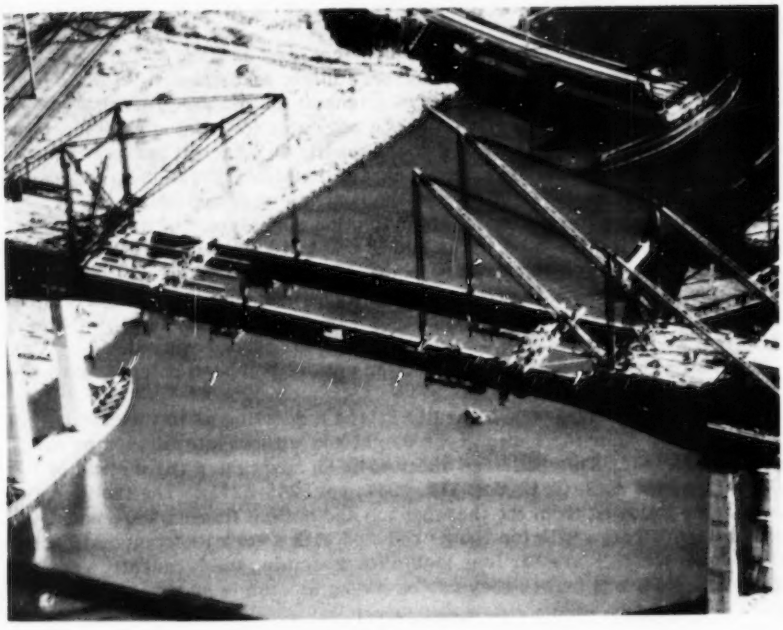


Figure 18. Final Splices Made

As the closing girders were entered, the splices at the north or fixed-shoe end were contacted, then pinned and bolted for the lower third. At the south end the cantilever girders were pushed out into contact by jacking horizontally at Pier 1S; the east girder 2-1/4" (which was close to the theoretical) and the other 3", with jack loads of 192,000# and 288,000# respectively. The motion over the shoe never exceeded the gain at the splice by 1/2 inch, meaning no excess deflection of the pier top. Both south splices came approximately 70% fair, and as soon as the lower third of the splice was pinned and bolted the load in the falls was gently lowered to make the remaining holes. The northeast splice was made by the time all the load in the south falls and north falls had been released, while the northwest splice was still only 70% fair. It was necessary to pick up the falsework bent 3NW two inches to make the top chord holes in this latter.

As the floor steel was thereafter being filled in, it was noted that the bottom flange holes in the girder at pier 1S were approaching the holes in the top shoe; when the distances between holes had diminished to 3/8" for the east girder and 7/8" for the west girder the horizontal jacks were used to line up the holes, and permanent stop plates were placed against the shoes and riveted. This jacking indicated roughly a friction coefficient of .03.

As falsework bent No. 4 had been effectively removed after traveler moved beyond Pier 1, the girders were raised over Pier 2 by raising the jacks under falsework bents No. 3; following up on bents No. 2, under the suspended span, to keep the pins in the center of the slotted holes and thus to avoid loading the links. The omitted rollers were then slid laterally into position. This jacking

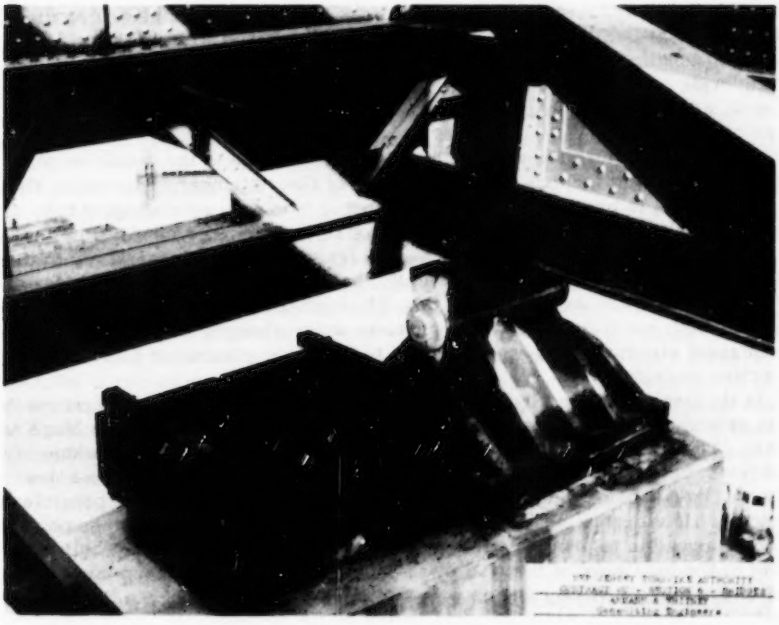


Figure 19. Shoe and Rollers on Pier 2S.

on bents No. 3 was of course against the elastic resistance of the completed continuous structure. Bents No. 3 were then lowered to load the rollers on Pier 2; bents No. 2 were lowered to move the end reaction of Spans 3 onto the links, and the steel was then completely "swung".

From erection of section 3SA, first element of the five main spans, to the final swinging, occupied 70 days.

The grade of the concrete roadway slabs was determined, throughout the bridge, by screed angles along their edges, riveted to the floor steel. Concreting on the five main spans had to proceed from the two ends during an interval in which the travelers were being dismantled and lowered, and the contour of the girders was irregular and daily changing. So, whereas on the approaches the screed angles had been adjusted on the basis of the pre-computed deflections from concrete weight, on the main spans they had to be set from the existing girder contours, anticipating that these would come correct when the travelers were down. This expedient was not later regretted.

Fabrication and erection were carried out by the Bethlehem Steel Company, division of Fabricated Steel Construction, of which Frank A. Strouce is General Manager. The advance engineering was directed by the author, assisted particularly by W. H. Jameson and George L. Gray. Erection was in charge of J. H. Wagner, General Manager of Erection, through E. L. Durkee, Engineer of Erection and George P. Adair, Manager of Erection, New York District.

In the effort to condense this description for purposes of oral presentation, some items of technically interesting information have been omitted from the foregoing. Using the above as a background, the more important of these are here appended.

Girders vary from 10-0-1/2" to about 11'9" for suspended side spans and from 13'3-1/2 to 21'-6-1/2 back to back angles for the main river crossing. Main material is silicon steel. Web varies in thickness from 9/16" to 3/4", stiffened longitudinally over main Piers 1S and 2S. Each flange is composed of 26" wide cover plates, side plates under four 8 x 8 x 1-1/8 angles and, at places, additional side plates over the web legs and between the outstanding legs of the angles.

An unusual feature for a through plate girder bridge is the provision made at each floorbeam for expansion of the pair of floor stringers adjacent to the main girders, to prevent large lateral bending moments developing in the floorbeam flanges. The stress deformation of the flange or chord while stringers remain of practically unchanged length, is a very real worry in the erection of long span bridges, especially the stringer closest to the girder or truss. During cantilever erection this differential is of course much larger than normal; but it is frequently a problem under changes of stress in the permanent structure. We were pleased to have this eliminated from our erection worries.

In the final state the girders are fixed at Piers 3 and Pier 1 - therefore it was necessary during erection to provide longitudinal stability at the hinge point. This had to be not only flexible vertically to allow for relative deflections, but also adjustable longitudinally to allow for making of the holes for the fixed shoes at Pier 1N at any expected temperature and for possible shop and field errors. The change in depth of the girders at the hinge point made it possible to provide a link detail which connected to a 3"eyebolt fastened to the top flange of the shallower suspended girder. Bending was taken out of the eyebolt by clip angles with slotted holes over end of pin through the eyebolt. This detail was also complicated by the necessity of running the traveler over it as erection proceeded.

At the hinge point it was also necessary to leave the bracing connections loose, leave loose all interlocking material, and raise the steel mall and curb sections of the main spans high enough to clear the adjacent sections. This being done to allow sufficient longitudinal clearance with the girders in their lowered position at Pier 2, enough permanent clearance was available between the girders at the hinge that no change in the girders themselves was necessary. Due to the different vertical curvature at the two link locations, it was calculated that the clearance between girders 3 and 2 would be reduced less at the south end than at the north end, when rotation would be introduced about Pier 1. Therefore the shoe at Pier 1S was made the temporary expansion shoe for closing the main span, while permanent connection of girder to shoe at Pier 1N would be made when first erected.

At Pier 2 the requirements were that (1) the girder should be not more than 17" low; (2) free longitudinal movement be permitted; (3) transverse motion be prevented and (4) it be possible to erect the rollers on the piers adjacent to the shoes at the same time the shoes were erected, and skid them sideways into place after the girders had been made continuous. In order to insert the rollers by skidding sideways the girders had to be jacked 6" above normal to clear the toothbars.

Final deflection calculations proved that we would have sufficient latitude for all conceivable errors by erecting the shoe and girder 16-1/2 inches low, leading the falsework bents 3, and varying the angle of the ends of the main-span cantilevers by jacking up or lowering on falsework bents 3 to match the ends of the closing girder. Omission of the 18 inch rollers left room to replace them with a greased plate, grooved top and bottom. The greasing allowed longitudinal motion, and the grooving engaged the ridges on the top shoe casting and on the base plate which normally prevent transverse motion of the rollers. Calculation were also made to ensure that with the girders finally continuous we would have sufficient jack capacity at bents 3 to raise the girders 6" above normal at Pier 2 without overstressing the girders.

The elevations of bents 2, 3 and 4 were so set that with the bearing at Pier 2, 16-1/2" low, the camber position of the girder at Pier 1 would be 1" high. This was done to allow for girder deflections and settlement, if any, of bent 4 under maximum load when picking girder section 1 (see Fig. 12). With this 1" allowance it was thought that, when girder section 1 rested on Pier 1, the splice just ahead of falsework 4 would be exactly lined up or the bottom splice holes would be fair with the top of the splice open, thus requiring jacking down at bents 3 and 4 to make the splice. Three of the four splices were made without jacking. The one that had to be jacked, however, was jacked up.

The positions of the cantilever ends of the girders were calculated for just prior to picking the closing girders, and for after picking the center girders (since the derricks were standing on these cantilever arms). It was hoped that a check on this could be obtained in the field. As a minimum it was prescribed that readings should be taken before picking the girders, to determine if the openings both top and bottom would be sufficient and the relative elevations of the girders be nearly right. As it turned out, only the minimum readings were made; and even that was with the booms of the north derrick working, with the load on both the anchor span and the cantilever increased to expedite erection, and with the falsework at bent 3N about 3 inches low. Comparing these measurements however with the computed theoretical for very similar loadings it was apparent that the closing girders could be entered and that bent 3N could be jacked up, if necessary, to close the splices. The reason for taking only the minimum readings was to speed erection, so that the

girders could be erected on the day planned; since conditions were nearly perfect, with the sky cloudy, temperature close to normal, and no wind. This last was important as there were few days at this time of year, October, that had not been windy.

It seems well in conclusion, to point out the advantages in both time and money saved, on the construction of the approaches of the Passaic River Bridge. by the contract requirements that the foundation contractor build a specified construction roadway. This roadway was called for under 34 of the approach spans, with a total length of approximately 4400 feet. These spans were over New Jersey meadow land which is usually under water most of the time. By specifying a satisfactory construction roadway, in this case approximately 113 feet wide, each of the superstructure contractors knew at the time of bidding just what the site conditions would be and how materials could be delivered. With a road of that width, deliveries could be made simultaneously to the foundation contractor and to the steel erector without causing delays to either. Both contractors had a suitable working platform for unloading and sorting their materials. Subsequently the slab contractor delivered his forms directly to the spans as required, and concrete to hoist-towers spaced along the edge of the superstructure, without interfering with steel erection. Later the painting contractor used the construction roadway. We believe the Turnpike Authority will also find this roadway useful throughout the life of the superstructure.

Omission of the construction roadway from the foundation contract would have meant that:

(a) The foundation contractor would have included in his bid the cost of a minimum road. Not knowing whether he could sell the use of this road to the steel contractor he could not allow for any salvage value.

(b) The steel contractor, knowing the foundation contractor might hold out for an exorbitant price for the use of the road, would probably have figured on running the 13,000 tons of steel out over the top. This is a slow and costly method, which furthermore slows up the slab contractor.

(c) The slab and painting contractors would also include additional delivery cost in their estimates.

The sum of all these individual allowances would, in our judgment, be greater that the actual cost of the construction roadway; without making any allowance for the time saved in completing the contract for a revenue-bond structure.